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Full-scale testing of a damaged reinforced concrete frame in fire

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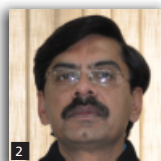
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Fires are a relatively likely event following earthquakes in urban locations and in general are an integral part of the emergency response strategies, which are focused on life safety in most developed economies. Similarly, building regulations in most countries require engineers to consider the effect of seismic and fire loading on structures to provide an adequate level of resistance to these hazards; however, this is only on a separate basis. To the authors' knowledge there are no current regulations that require buildings to consider these hazards in a sequential manner to quantify the compound loading and design for the required resistance. This paper provides a first and early report from a novel set of tests on a full-scale reinforced concrete frame subjected to simulated earthquake and fire loads. The results from the first test indicate that the test frame could withstand a pre-damage corresponding to a seismic performance level and subsequent 1 h fire exposure without collapse. Important observations have been made about the development of temperatures and displacements in the various structural elements during the mechanical loading and subsequent fire excursion.

1. Introduction

The risk of fires in the aftermath of earthquakes is well known. The fires following the 1906 San Francisco and the 1923 Tokyo earthquakes led to major conflagrations and widespread devastation, resulting in far greater damage than caused by the original shaking. Fortunately, the scale of those events has not been repeated; however, there have been many major earth-

quakes that have been followed by fires. Nearly all major Californian earthquakes have been followed by multiple ignitions, most notably the 1971 San Fernando and 1994 Northridge earthquakes, which were both followed by over 100 ignitions. The 1995 Hanshin (Kobe) earthquake was also followed by over 100 ignitions in Kobe city and a similar number of fires in other cities in a highly populated area (over

2 million people), and several conflagrations developed. Scawthorn *et al.* (2005) provide a relatively comprehensive treatment of the post-earthquake fires from an emergency response, societal preparedness and disaster mitigation point of view and include discussions of the major historical fire-following-earthquake (FFE) events.

Another fact that emerges rather starkly from the study of FFE events is that the risk of FFE is very non-uniform. Many recent earthquakes were not followed by widespread fire events, for example 1999 Izmit (Turkey) (although a number of crude and naphtha tanks burned), 2001 Gujarat (India), 2005 Kashmir (Pakistan and India) and 2008 Wenchuan (China) earthquakes were not followed by significant fire events. The level of urbanisation and industrialisation is an obvious factor which possibly explains this anomaly (most certainly for the relatively remote and backward mountainous regions of Kashmir – even here, however, the main market in the town of Uri suffered a major fire following the earthquake, which caused extensive damage). If urbanisation (and concomitant density of gas, fuel and electrical supply networks) is indeed one of the key reasons, the risk of fire after earthquakes must then be considered as a rapidly increasing risk to life, livelihoods and to the sustainability of growth and development in some of the world's most densely populated regions. With an increasing integration of the world economy, major disasters of the future could have repercussions far beyond the local region. FFE events have the potential to create such disasters and should certainly be considered in the overall disaster mitigation strategies by governments and agencies with such a remit. Considerable new research effort is required to properly address the challenge posed by FFE events, some of which are discussed by Botting (1998), Cousins *et al.* (1991), Robertson and Mehaffey (2000) and Usmani (2008).

This paper will describe the progress on a collaborative project involving a number of institutions in India and the UK (2008–2011). The key aim of this project has been to carry out possibly the first ever large-scale tests to understand the behaviour of damaged reinforced concrete (RC) frames in fire. Considerable new understanding was gained on the behaviour of steel-framed composite structures in fire from the Cardington test in the mid-1990s (British Steel, 1999) and a large collaborative project on

the computational modelling of the tests (Lowes *et al.*, 2004). It is expected that these tests will help generate similarly useful information on the behaviour of damaged and undamaged RC structures subjected to fire.

After a great deal of preparation, these tests began in February 2011 and the full test programme is intended to end by summer 2011. The programme is expected to produce a great deal of new information and experimental data on the behaviour of damaged RC structures in fire, which should be of considerable international interest to researchers and practitioners in structural engineering. The project team has, therefore, set up a round-robin modelling exercise. The exercise requires participants to predict aspects of behaviour witnessed in the test programme described above. It is hoped that all those who attempt to model the test will in due course also participate in a workshop to be organised in India and present their results (which will be published in a compendium). The full experimental results will be made available to all participants at the workshop (and published on the workshop webpage).

2. The plan for large-scale testing of damaged RC frames

Tests were planned on a number of identical RC frames consisting of four columns (3 m apart in plan) supporting four beams and a slab, all monolithically constructed, at a dedicated testing facility on the campus of the Indian Institute of Technology (IIT), Roorkee in India. These frames were proposed to be subjected to cyclic quasi-static loading against a reaction wall, which would provide a reasonable simulation of the damage expected to occur under real seismic loads. Table 1 shows the planned tests. The first frame test was aimed to subject the frame to displacement beyond the peak lateral force. The frame would then be exposed to fire capable of attaining a temperature up to 1000°C or beyond. The frame would subsequently be subjected to further loading to evaluate its residual capacity in each case. While the second frame test aims to evaluate the fire damage on an undamaged frame, the third test is planned to introduce an 'intermediate' damage level that would correspond to a certain (x) percentage of the horizontal slab displacement achieved at the peak lateral force obtained in the first test.

The first test on the frame has already been carried out as planned.

Test no.	Simulated seismic damage	Fire loading	Aftermath
1	Displacement beyond peak lateral force	900–1000°C ^a	Residual lateral capacity test ^a
2	None	900–1000°C for 1 h	Residual lateral capacity test
3	'Intermediate' damage ($x\%$ of the displacement corresponding to peak lateral force)	900–1000°C for 1 h	Residual lateral capacity test

^a For as long as considered safe.

Table 1. Frame tests planned

Initially, the frame was subjected to a simulated seismic damage. The initial seismic damage was achieved by inducing a pre-planned lateral displacement through applying lateral cyclic load. The planned lateral displacement corresponds to the life safety

level of FEMA 356:2000 (FEMA, 2000). Figure 1 shows a schematic diagram of the front elevation of the test frame set-up with key dimensions. The frame was tested on a 1.2 m thick raft of dimensions 6.75 m \times 8.55 m (Figure 1a). The lateral load was

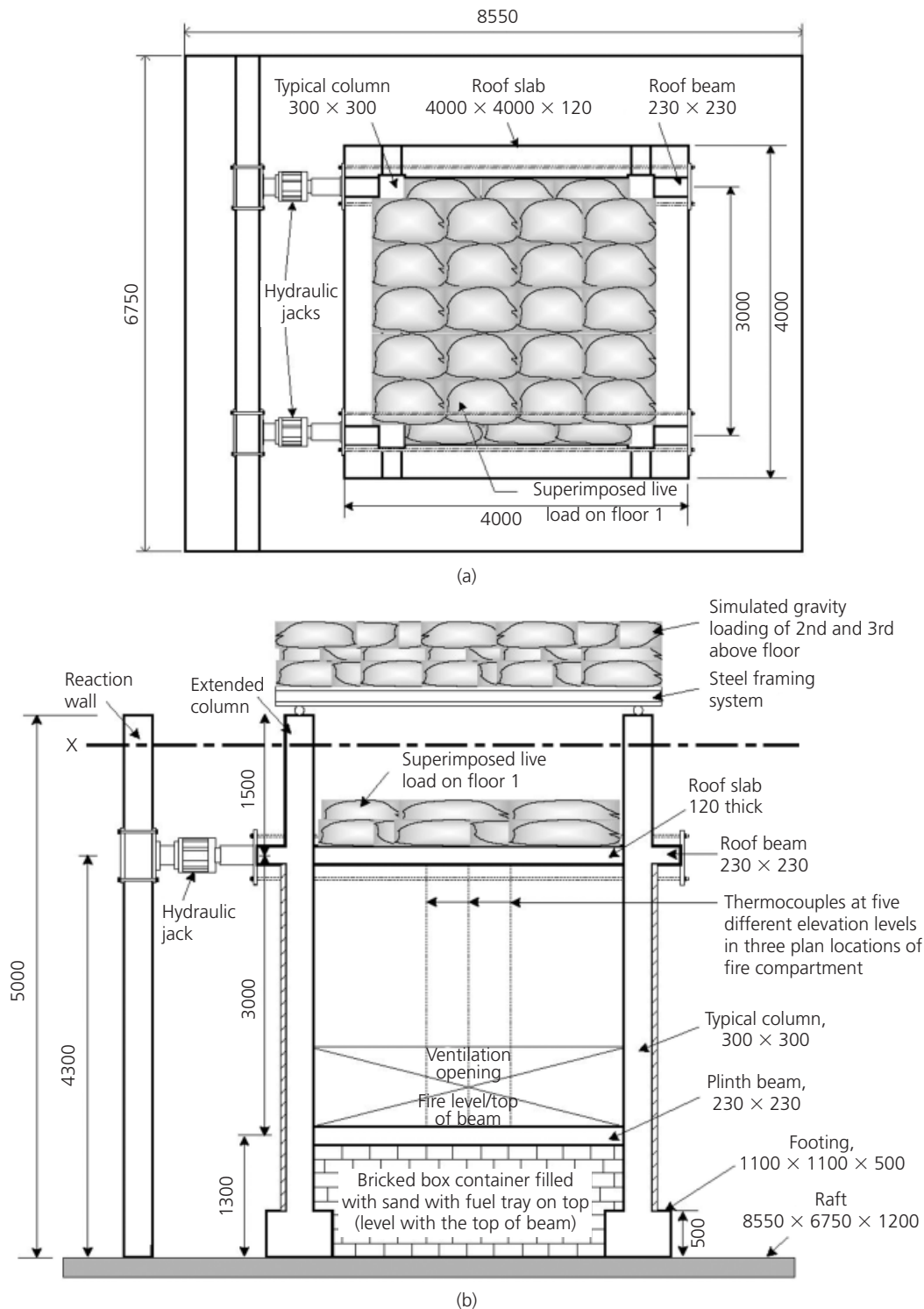


Figure 1. Schematic view of the test set-up: (a) plan of the test set-up; (b) elevation of the test set-up (all dimensions in mm)

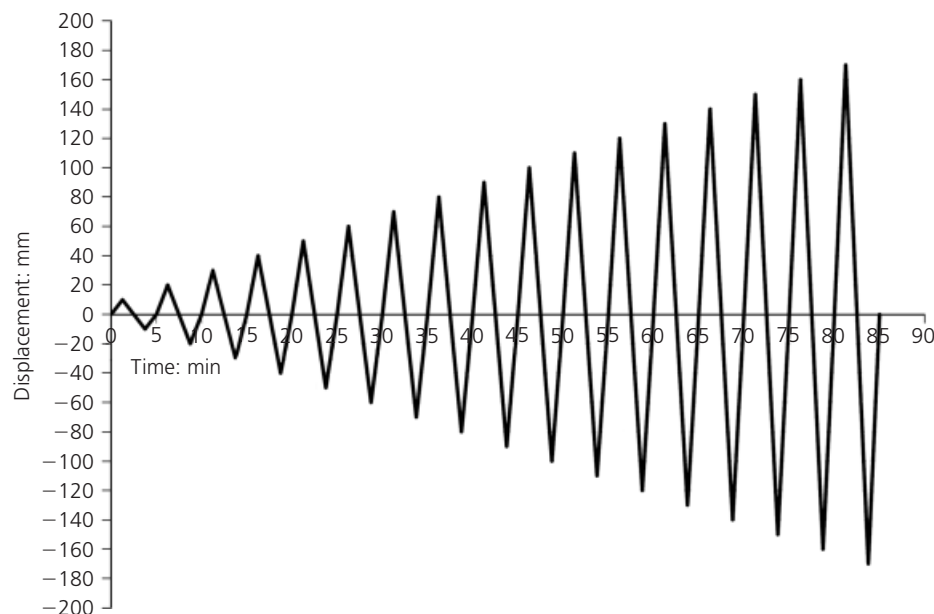


Figure 2. Incremental cyclic loading of the frame; roof displacement plotted against time

applied through jacks, which were mounted on a vertical reaction wall. Subsequently, the frame was exposed to fire of 1 h duration. Detailed thermal and mechanical histories were recorded during the initial loading and subsequent fire. Thermocouples embedded at three different locations in plan and at five layers along the section of the beams recorded the temperature history. The displacements were also measured by erecting a secondary steel frame around the test frame, during both the initial damage-inducing phase and the fire phase. Particle image velocimetry (PIV) equipment (high-resolution digital camera and image analysis software) was also used to create an independent set of data for comparison with the mechanical data using traditional methods.

3. Fire testing

The fire compartment of size $3\text{ m} \times 3\text{ m} \times 3\text{ m}$ was constructed using detachable panels made of fire-proof materials commonly used in brick kilns in India. This was expected to allow repeated use of the panels for the whole testing programme and beyond. The fire was continuously fed by a 1 m square tray of kerosene in the compartment with a 1 m high opening along the full length of the wall at the bottom of one side (Figure 1). To maintain a post flashover temperature of $900\text{--}1000^\circ\text{C}$ the peak burning rate for the chosen opening configuration was approximately $0.117\text{ kg/m}^2\text{s}$. This required a peak flow rate of kerosene into the tray of $1.43 \times 10^{-4}\text{ m}^3/\text{s}$ which was maintained using a fixed head. About 0.51 m^3 of kerosene was required for maintaining the post-flashover temperatures within the above range for 1 h. The chosen configuration was designed to achieve flashover within 5 min.

Thermal instrumentation for the compartment consisted of three thermocouple trees in the fire compartment to capture the gas

temperature history inside the compartment during the fire testing phase. Adequate numbers of thermocouples were also embedded in the structural members to obtain detailed structural temperature evolution for the whole heating and cooling cycle. A number of mock fire tests were carried out at the location of the test to ensure that the expected fire behaviour was achieved and was repeatable. The first mock test was carried out in July 2009, and did not succeed, as the brickwork walls were very damp because of rain, so much of the radiant heat from the fire was absorbed by the wall, leading to an inordinate delay in flashover and low peak temperatures. The test was repeated in November 2009 and this time the results were as expected, as shown in Figure 3. This, however, was not expected to be an issue for the actual tests as the compartment was made up of waterproof insulating panels.

4. Computational modelling

The frame was designed as part of a four-storey building frame located in seismic zone IV according to Indian Standard IS 1893: Part 1 (BIS, 2002). India has been divided into four earthquake-prone zones: Zone II–Zone V, based on their occurrence. Zone IV corresponds to a severe intensity zone prone to major property damage and a zone factor of 0.24 is considered. The beam and column reinforcement detail obtained from the first design cycle is shown in Figure 4. The design was also checked against Eurocode 8 (CEN, 1998) and found to be sufficiently ductile to withstand the assumed earthquake loading.

To ensure that the design did indeed provide adequate ductility, a number of different numerical models of the test frame were developed to investigate the expected behaviour of the frame under the imposed quasi-static displacements (Figure 2).

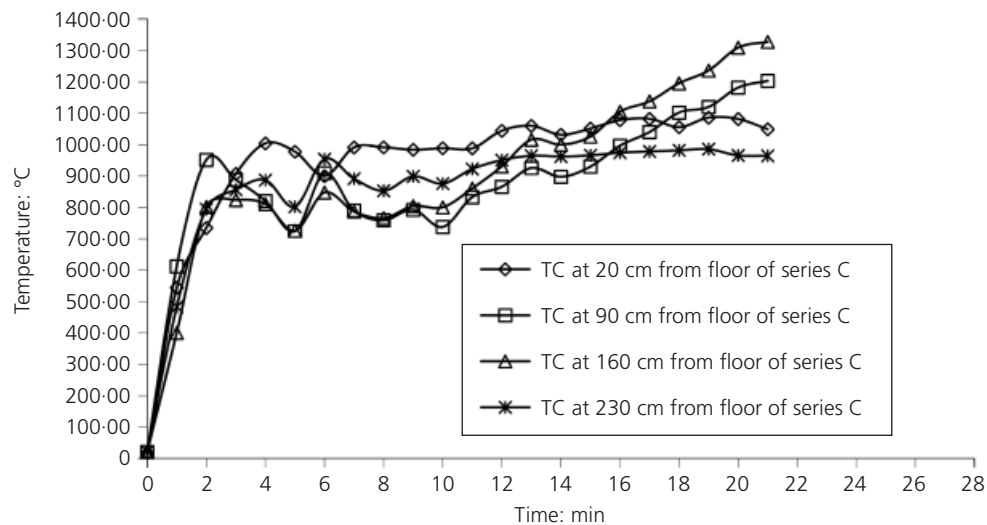


Figure 3. Temperatures inside the fire compartment (near the centre of the back wall opposite to opening)

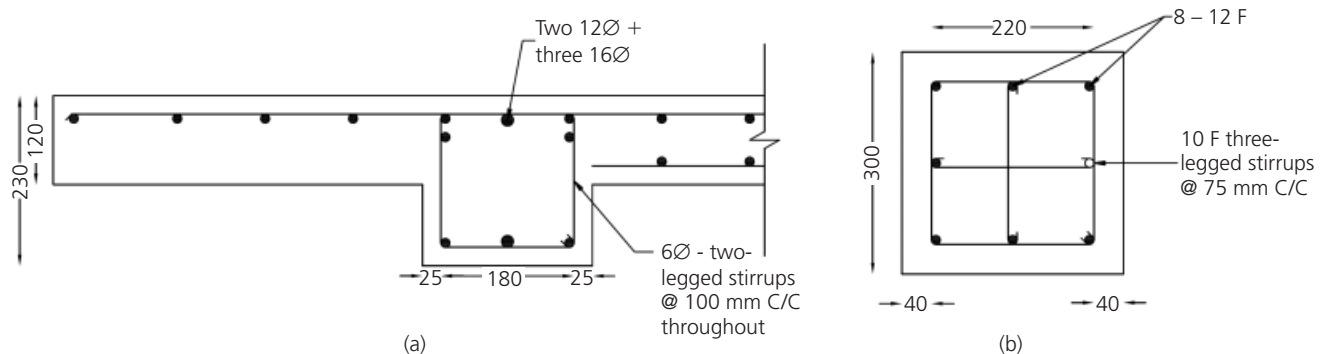


Figure 4. Details of (a) beam and (b) column reinforcement (dimensions in mm)

Although not explicitly stated in the standard, it was expected that if the code-based design recommendations were followed, strong-column weak-beam type behaviour would be obtained, that is the first hinge would form in the beams. A simple plastic analysis showed that this was not the case for the one-storey test structure and this was later confirmed by more detailed finite-element frame analyses, as shown in Figure 5.

There are two main reasons for this discrepancy: first, the design is based on a four-storey structure where the beam moments at the joints are balanced by the sum of moments in two columns (upper and lower), while for the single-bay single-storey structure the moments in the beam and column must be the same; second, frame analysis, which is the usual method for analysing structures under earthquake loads, does not fully account for the significant additional moment capacity from the monolithic slab acting compositely with the beam.

This issue was discussed in detail between the UK and Indian

teams and it was decided to revise the design to produce a more desirable ‘seismic’ behaviour and ensure that first hinge formation occurs in the beam. The reinforcement in the column was increased (to eight 20 mm dia. bars) and that in the beam was decreased (to three 16 mm dia. bars top and bottom). Now the results from a single-storey frame model analysis in the SAP2000 software package (SAP) showed that first hinges formed in the beams; however, a more detailed Abaqus finite-element model (using brick elements for columns, beams and slabs) was still inconclusive, with plastification starting at joints (as shown in Figure 6) where the beam is composite with the slab (lower beams without slabs clearly formed hinges first). The revised design was, however, adopted.

5. Peak load and displacement

The computational models also allowed estimates of peak lateral load and displacement curves to be made. The peak load from a plastic analysis calculation for the original frame (before the modification mentioned above) was found to be approximately 140 kN. Figure 7 shows the results from ‘pushover’ type analyses

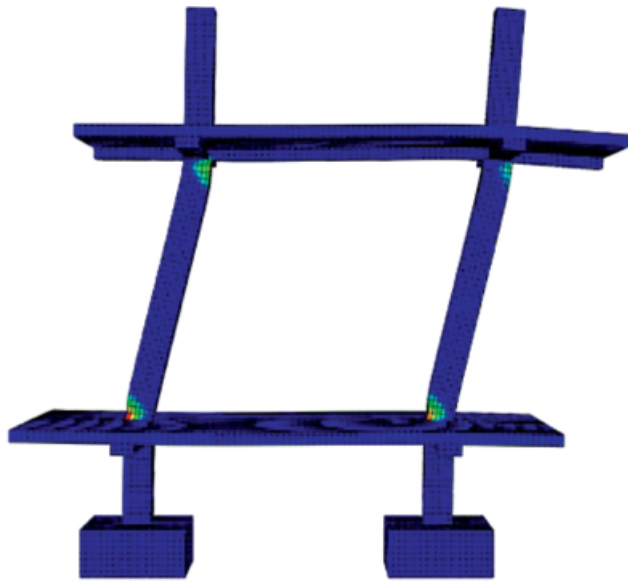


Figure 5. Finite-element model of the frame showing hinges in columns

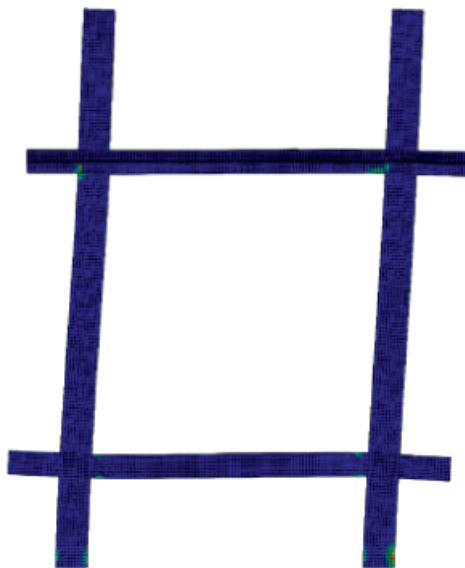


Figure 6. Finite-element model of the modified RC frame (with stronger columns and weaker beams)

carried out using two different software packages and using different types of models based on the modified design. Figure 7(a) shows the results from a quasi-static non-linear SAP frame model and Figure 7(b) shows the results from two non-linear dynamic finite-element models (using Abaqus). The two Abaqus analyses (with the same data) were carried out using the ‘implicit’ dynamic procedure and the conditionally stable ‘explicit’ dynamic procedure, both of which produced similar results (with the explicit procedure taking significantly large computing

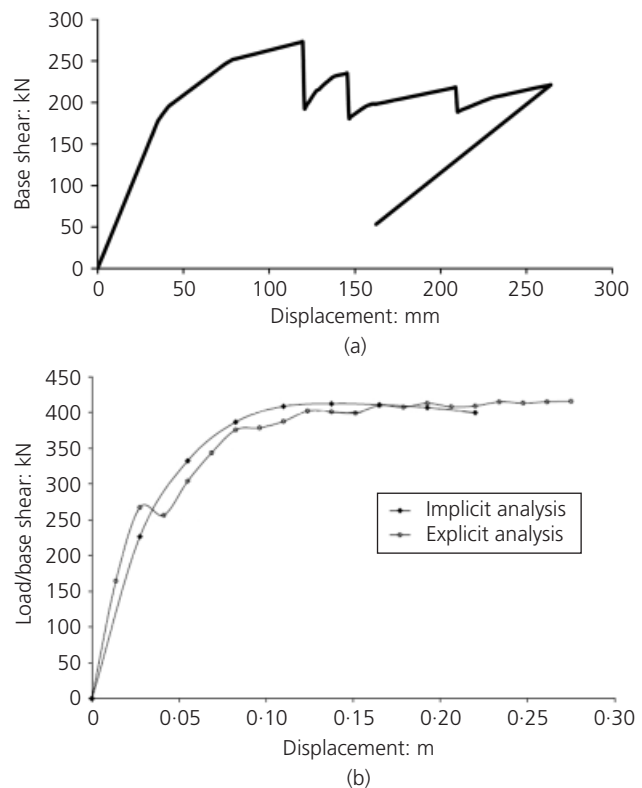


Figure 7. Load–displacement curves from (a) frame model and (b) detailed finite-element model

time). The Abaqus models seem to overestimate the peak load considerably; however, at a displacement of roughly 50 mm the two models are somewhat in agreement (200 kN and 260 kN); the displacements at peak load are also similar (~100 mm). It is likely that the Abaqus model, constructed of brick elements, is over-stiff (despite using non-conforming elements and a relatively fine mesh). An Abaqus beam and shell model provided results comparable to the SAP model. These two preliminary analyses were primarily carried out to obtain upper and lower bound estimates of the frame load displacement response for planning the experiment properly and to obtain useful data on the capacity and stroke lengths of the loading jacks required to carry out the damage-inducing cyclic loading for the frame. A more detailed predictive analysis is reported in the next section.

6. Predictive models

A number of further analyses are also being carried out to predict the actual test behaviour of the frame. This is being done using software such as Abaqus and OpenSees (modified by the University of Edinburgh team to include thermal loading). These analyses are simulating the cyclic loading procedure, where the displacements will be applied in both directions in increasingly larger increments (Figure 2) until the displacement corresponding to the peak load has been achieved. The simulations are then continued to the fire loading phase based on the temperatures

obtained in the mock tests (Figure 3). This work will be reported in much more detail in a number of separate reports and papers; however, brief results from one of the predictive models (using OpenSees) are presented here.

The fire loading was based on a considerable simplification of the temperature field shown in Figure 3. The fire was assumed to be represented by a constant temperature of 1000°C at the boundaries of the structural members applied for 1 h. A one-dimensional heat transfer analysis was carried out to determine the temperature evolution in the structural members, which was then used to determine the mechanical response (after the damage-inducing cycle).

Figure 8 shows the lateral (or horizontal) displacement of the frame through five cycles of loading, which produces a peak load of roughly 250 kN, corresponding to a displacement of approximately 76 mm, which agrees reasonably well with the SAP ‘pushover’ analysis of Figure 7. Figure 8 also shows that a permanent ‘drift’ of approximately 19 mm (leftward) occurred at the end of the cyclic loading.

Figure 9(a) shows the vertical displacement of the mid-span of the top beam (node 3) during the fire loading phase, and Figure 9(b) shows the horizontal displacement of the end nodes (nodes 1 and 2) of the top beam during the fire phase (both continuing from the permanent residual displacements at the end of the cyclic displacement). Both figures are plotted to start from the point where the nodes were located at the end of the gravity and cyclic loading stages. Therefore, node 3, in Figure 9(a), was displaced in the positive direction (upward) and nodes 1 and 2, in Figure 9(b), were displaced in the negative direction (leftward) at the end of the cyclic loading. The initial displacement of node 1 in Figure 9(b) is the same as the drift mentioned for Figure 8, above. An interesting and counterintuitive feature of behaviour from this analysis is that during the early phase of the fire, the

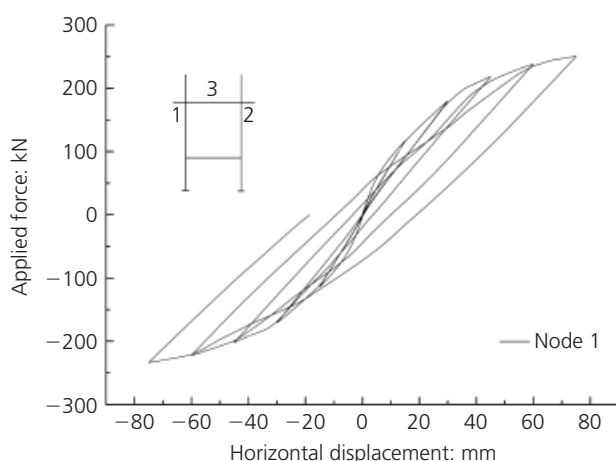


Figure 8. Horizontal displacement of node 1 under quasi-static cyclic loading

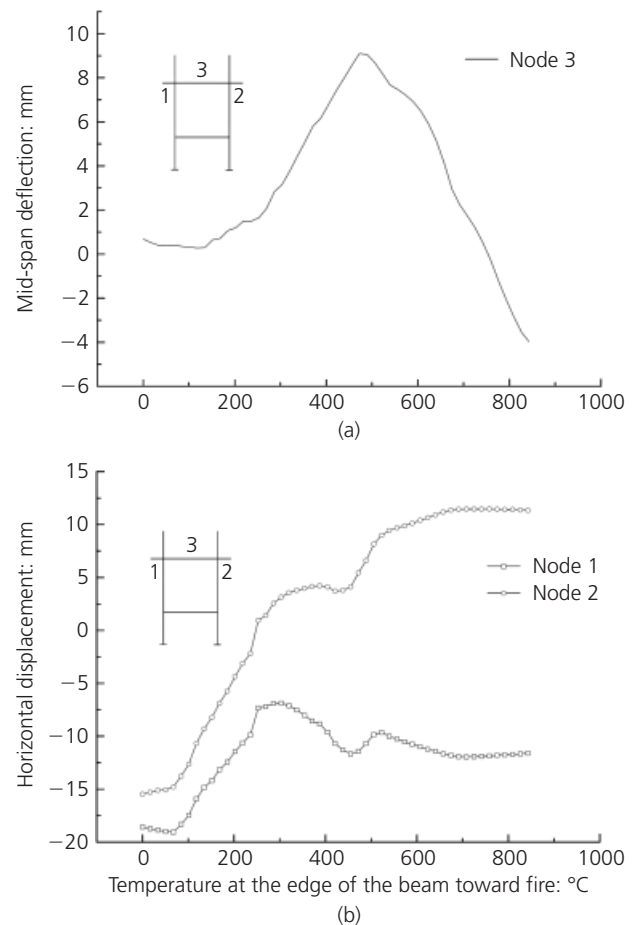


Figure 9. Displacements during the fire loading phase: (a) node 3 at mid-span of top beam; (b) at nodes 1 and 2

frame as a whole moves towards becoming more ‘upright’. The permanent lateral ‘drift’ at the end of the cyclic loading of about 19 mm is initially reduced to about 7 mm when the exposed surface of the beam reaches approximately 300°C, which suggests a ‘stiffening’ of the frame. After this, however, the drift begins to increase again, but plateaus out by the end of the heating.

7. Preliminary test results

This section presents some key results of the first test conducted in March 2011. A pre-planned mechanical damage in the form of 2% of the storey height (FEMA 356, FEMA, 2000) was introduced in the frame by applying lateral quasi-static cyclic loading. Seven incremental lateral displacement cycles of equal push and pull were applied using the loading jacks, as explained earlier. Figure 10 shows the recorded lateral load–displacement response of the test frame. A maximum displacement of 95 mm in push and 85 mm in pull was registered, corresponding to load levels 316 kN and 267 kN, respectively. After each cycle of push and pull, a careful visual inspection was carried out to locate cracks and spalls, if any. All the cracks were graded with permanent markers. No cracks were noticed up to a lateral displacement of

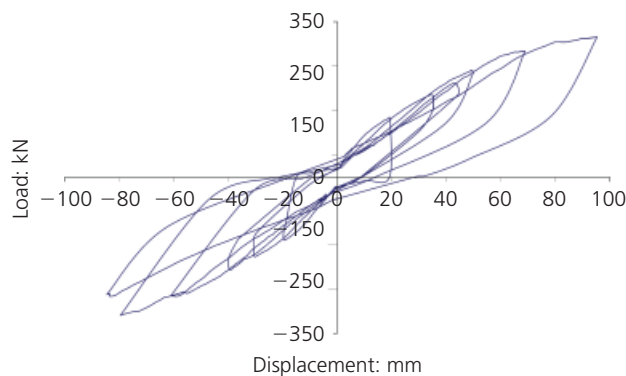


Figure 10. Test load–displacement response of frame

20 mm. However, first visible cracks were observed at the ends of the roof beams aligned parallel to the direction of loading after the third cycle of push and pull and at a displacement of 30 mm. In subsequent cycles, these cracks propagated and further cracks were observed in beams and columns as well. The cracks were observed to be more pronounced at the beam–column joints. However, the frame remained structurally stable after the initial mechanical test.

Although the permanent residual displacement after the cyclic loading phase matched well with the OpenSees prediction of approximately 20 mm, the ‘pinching’ effect clearly visible in the test results was not predicted by the computer model. The model was rerun with an OpenSees ‘pinching material’ (Lowes *et al.*, 2004) used for all elements adjacent to the joints. This produced a more realistic comparison with the test, as shown in Figure 11. Figure 12 shows exaggerated displaced shapes of the frame after (a) cyclic loading and (b) fire loading. Figure 12 shows the frame displaced shapes at the end of the cyclic displacements and at the end of fire. The displaced shape (b) is more reliable than (a), as the fire-induced displacements cannot be modelled properly by a two-dimensional model (more comprehensive three-dimensional models are currently being developed and will be reported in due course). Nevertheless, Figure 12 clearly shows that the frame as a whole does move towards becoming more ‘upright’ (as predicted by the previous model shown in Figure 9) and some of the drift is

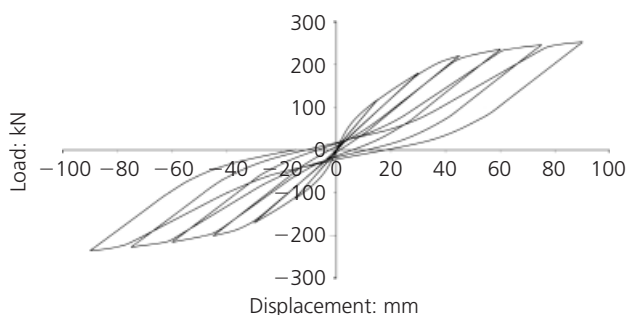
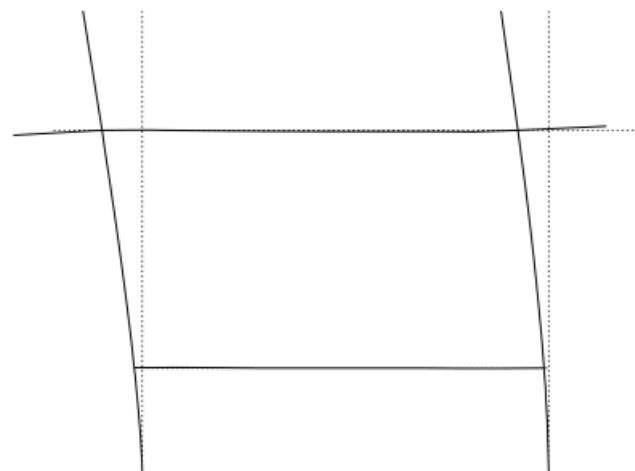
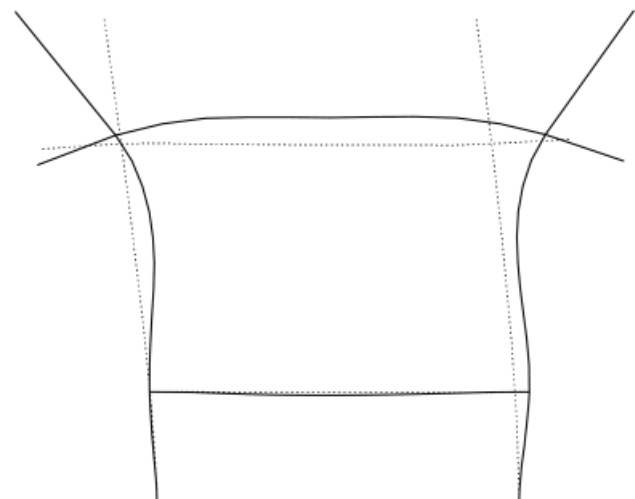


Figure 11. Modelled load displacement using ‘pinching material’ (OpenSees analysis)



(a)



(b)

Figure 12. Frame displaced shapes (magnified 20 times): (a) at the end of the cyclic displacements; (b) at the end of fire

recovered. However, it is notable that the drift recovery is much less in this model (more consistent with the test results), suggesting that hysteretic damage has been relatively more accurately modelled by using the pinching material. Figure 12(b) also clearly shows the effect of thermal expansion. Figure 13(b) shows the lateral displacement of nodes 1 and 2 of the frame for the updated model (corresponding to Figure 9(b)). The increasing relative displacement between nodes 1 and 2 is because of thermal expansion.

Four days after the cyclic loading, the pre-damaged test frame was exposed to fire. The fire test was performed for the planned duration of 1 h. The desired temperature–time behaviour, as achieved in mock fire tests earlier (Figure 3), could be obtained precisely, and full-blown fire with flashover was attained within 8–10 min. The sound of concrete spalling off from the roof slab

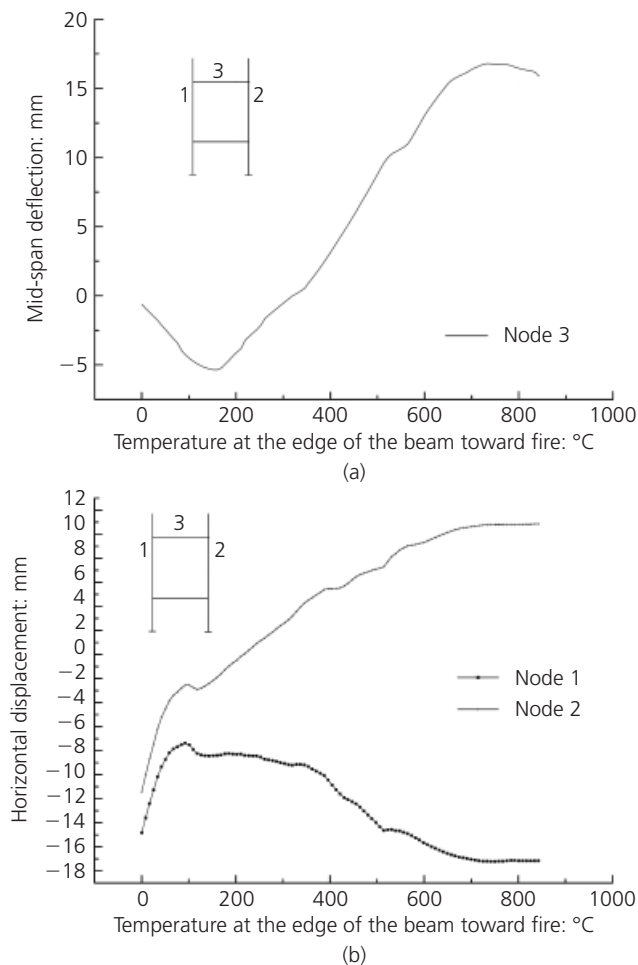


Figure 13. Displacements during the fire loading phase for the updated model: (a) node 3 at mid-span of top beam and (b) at nodes 1 and 2

could be heard clearly after 5 min from initial ignition, continuing for another 15 min. The spalling sounds corresponded to temperatures between 300 and 400°C. Later, the spalling of concrete was observed in beams and columns as well. The geo-PIV system did not perform well during the fire test owing to high temperatures and much reduced visibility because of the smoke. A detailed visual inspection of the frame was undertaken the next day after the frame had cooled down. Considerable damage was noticed in the slab in terms of spalling of concrete and resultant exposure of reinforcement. A maximum fire-induced vertical displacement of 46 mm was recorded in the roof slab. The beams and columns suffered considerable spalling on exposed surfaces, but only at a few locations; however, widespread cracks could be clearly marked in these elements. An attempt was made to drill a core for the purpose of conducting a core compression non-destructive test; this was, however, rendered impossible by disintegration of concrete all along the exposed surface. The frame was still able to withstand the 1 h fire exposure without losing structural integrity in terms of complete collapse. Temperature–time curves

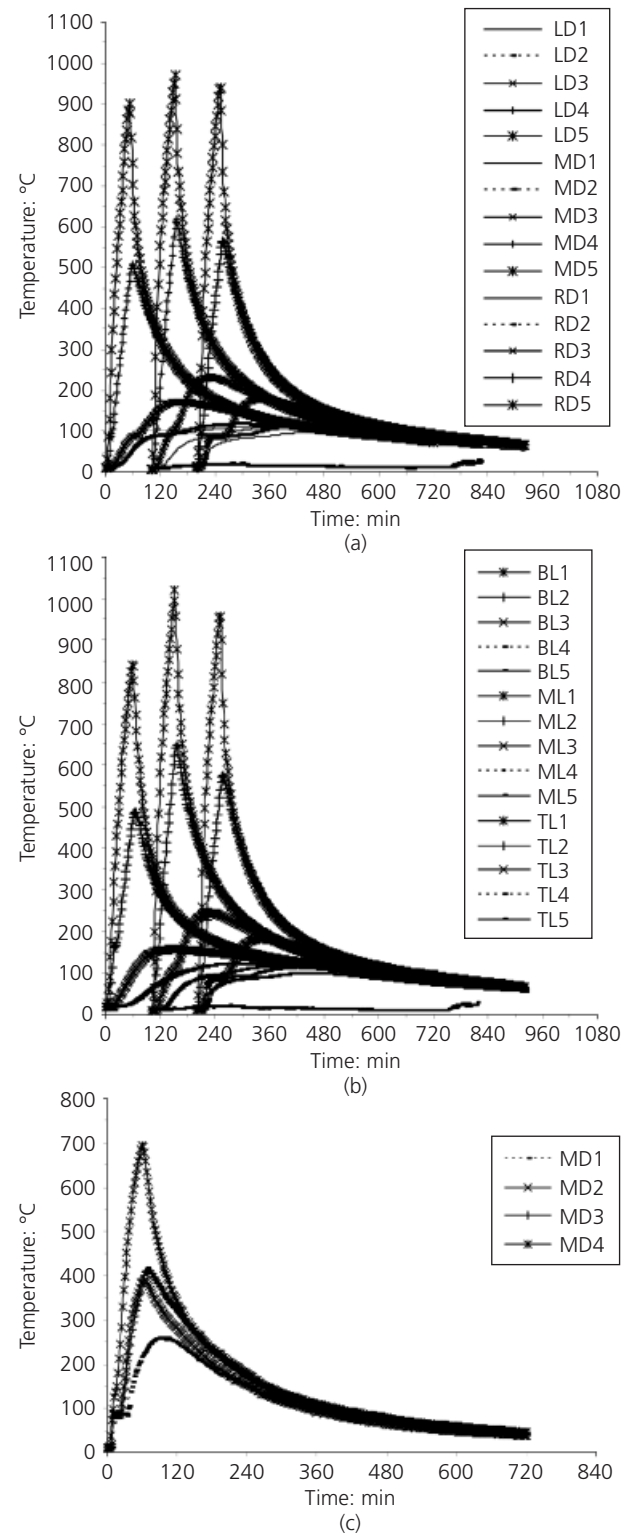


Figure 14. Temperature–time curves for: (a) typical beam (near joints and centre); (b) typical column (at ends and centre); (c) centre of slab (B – bottom of the column; T – top of the column; R – right of the beam near the joint; L – left of the beam near the joint; M – middle/centre of the beam; D1–D5 – depths at each location)

for typical members of the test frame are shown in Figures 14(a)–14(c). Maximum temperatures were recorded at the centre of the beam (972°C) and column (1002°C). Figure 14 also shows the thermal gradients existing in the beams, columns and slab by showing temperatures in deeper layers of concrete against the hottest (surface) temperature. Figure 15(a) shows the frame after pre-damage attributable to mechanical loading, and Figures 15(b) and 15(c) show the frame subjected to fire and its condition after the fire, respectively. Figure 16 and Figure 17 show typical crack patterns as noticed on columns and beams, respectively, after seven cycles of mechanical loading. Cracks were marked on the members after each cycle of displacement to highlight their initiation and propagation.

8. Conclusion

This study provides a first and early report from a novel set of tests never previously attempted. The main aim of the paper is to make the structural engineering community aware of this programme and draw their attention to this complex and potentially destructive problem. This paper presents the results of the first of the series of tests planned to be conducted under the test programme. Further tests are planned for summer and autumn of 2012. The data from all the tests will be carefully analysed, including comparisons with computational models, and will be reported in detail in reports and technical articles submitted to appropriate symposia and journals.

The results from the first test indicate that the test frame could withstand the mechanical damage and subsequent fire without collapse. As expected (based on the design modifications) the damage during initial mechanical loading phase was first noted in beams rather than in columns. The observed load–displacement response was predicted reasonably well by OpenSees and SAP computer models in terms of the peak load and the corresponding displacement for the initial cyclic loading phase. The permanent residual displacement of approximately 20 mm after the cyclic loading phase also seems to match well with the OpenSees prediction. The ‘pinching’ effect clearly visible in the test results was not predicted by the first OpenSees model; however, attributing OpenSees ‘pinching’ material to elements next to the joints produced an acceptable pinching response with the same drift at the end of the cyclic loading analysis.

One of the key lessons that has been learnt so far is perhaps typical of any large-scale testing programme, namely the unpredictability of the whole process. It is difficult to foresee all the problems that may occur in advance.

Acknowledgements

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Figure 15. (a) Pre-loaded frame before fire; (b) fire test in progress; (c) fire-damaged frame after test

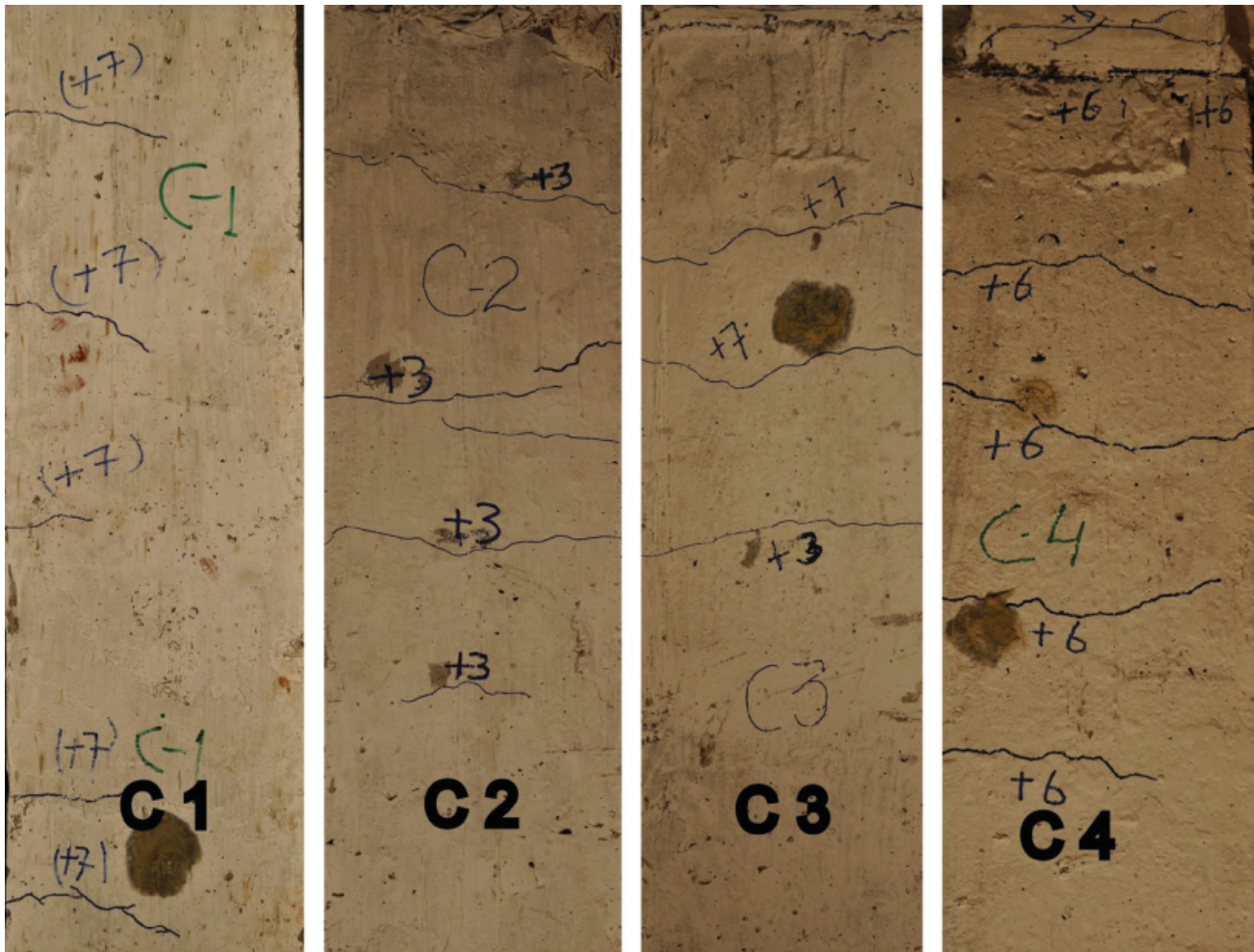


Figure 16. Crack patterns in columns attributable to pre-loading

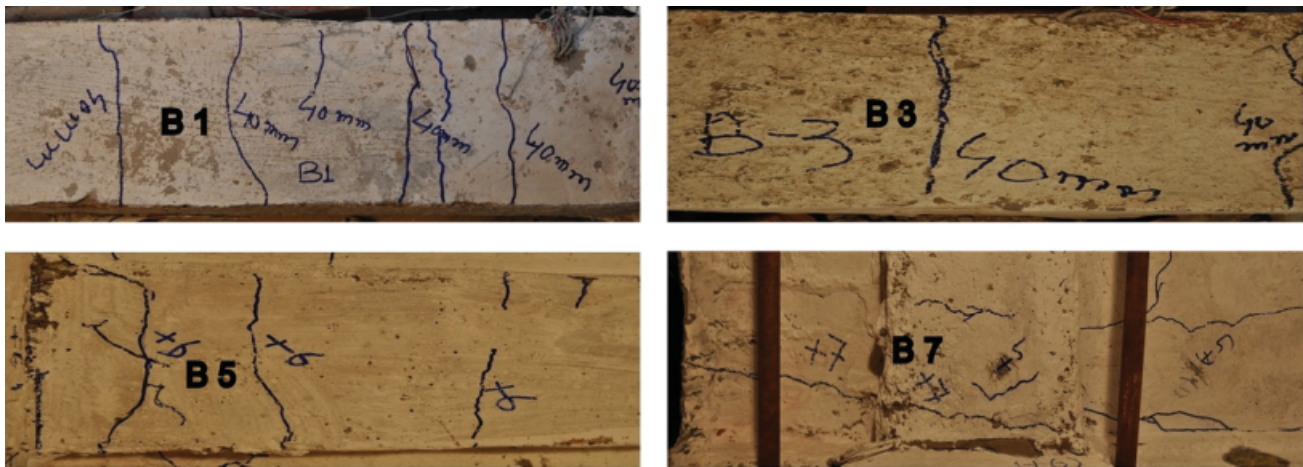


Figure 17. Crack patterns in beams attributable to pre-loading
(+3, +4, +6 and +7 markings on the beams indicate cracks
initiated at displacements of 30 mm, 40 mm, 60 mm and 70 mm,
respectively)

grateful to the OpenSees team at UC Berkeley and PEER for maintaining OpenSees and responding helpfully to their many questions.

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